Behaviour of Full-Scale Reinforced Concrete Frame Under Simulated Post-Earthquake Fire

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SUMMARY:
This paper provides an experimental insight of a single storey sub-assemblage, which is part of a G + 3 floors RC building subjected to simulated post-earthquake fire. In first part of the experimental program, a pre-decided initial damage was induced to the test frame through cyclic lateral loads in quasi-static. While in the second part, a compartment fire of 60 minutes duration was applied to the pre-damaged test frame for simulating the post earthquake fire scenario. The development of cracks and their patterns were recorded during each push and pull phase of cyclic lateral loading. Recorded strain data and the first observed cracks depict that the yielding of rebars at roof level in beam occurred prior to the yielding of the rebars in the column. Subsequently, the pre-damaged test frame was subjected to a compartment fire. Thermal and displacement histories of various components were obtained during the fire test.

Keywords: Post earthquake fire, reinforced concrete frame, performance

1. INTRODUCTION

Earthquakes are a great threat to human civilization located particularly in the urban areas and in several cases earthquake incidents are followed by fires which may further enhance the damages than the earthquake itself. Normally, reinforced concrete frame structures are designed to have resistance against an expected level of earthquake, and adequate fire resistance, with a consideration of occurrence of these two events separately. Buildings are usually designed without consideration of fire after the earthquake. The lack of sequential loading consideration in the design of RC structures for fire following earthquake may result in a catastrophe. The impact of fire following earthquake on the buildings and urban environment were analysed in detail [Scawthorn, et al. 2005] [Botting, 1998]. Past fire following earthquakes (FFE) accidents, viz.,1906 San Francisco, 1923 Tokyo, 1989 Loma Prieta, 1994 Northridge and 1995 Kobe earthquakes have demonstrated the importance of a comprehensive design requirement. Also, tragic event like collapse of World Trade Centre on September 11, 2001 [Mousavi, et al. 2008] [Yassin, et al. 2008] pronounce the need for design guidelines for accidental fire in structures. The fire performance of the structures significantly reduces during fire accidents. Such scenario creates a hazardous threat to structural integrity of buildings and proves to be detrimental to life safety of the occupants and rescue workers. Very few investigations on the building performance under combined action of both events are available for general practices in designing of structures. Also, the current design codes never practice to consider two extreme events to occur consecutively on a structure. Besides satisfying the structural design requirements for gravity loads including seismic and fire loads, buildings should be designed to withstand the FFE events for certain minimum duration of time, which is critical for the safe evacuation of the buildings. In present testing, a roof level displacement corresponding to the life safety level of structural performance was initially imparted through simulated cyclic quasi-static lateral loading to the test frame. This mechanical loading to the RC frame was followed by a compartment fire of one hour duration to simulate the fire following earthquake.

2. EXPERIMENTAL PROGRAM

2.1. Construction of RC Frame

A typical one-storey reinforced concrete moment resisting frame sub-assemblage was considered for the present study, which is extracted from a RC structure of G + 3 floors. The elevation and the plan of the frame were shown in Figure 1. The frame was designed for combined gravity and lateral loads in accordance with Indian Standard Code, and their members were proportioned and detailed according to IS 13290-1992 [IS 1893 (Part 1):2002] [IS 13920:2000]. All the elements of the test frame were monolithically cast and the columns were fixed in the RC raft foundation. The mixture proportions of concrete Grade M30 was used [IS 456:2000]. The simulated gravity loads (dead load + 25 % of imposed loads) of the upper floors were applied at the top of the columns of the test frame through the grillage system as shown in Figure 2, as the test frame is a part of RC building of G + 3 floors [IS1893 (Part 1):2002]. A pre-defined damage was induced with quasi-static lateral loads to simulate the realistic fire following earthquake.

2.2. Instrumentation

In experimental program, the RC frame sub-assemblage was first pre-damaged by the application of simulated equivalent lateral cyclic loading, after which the fire test was conducted on the pre-damaged frame. Hence, an exhaustive instrumentation was planned to log-in the data obtained during the process. The critical locations were first identified for the measurements of different parameters and then instrumented for the same. The strain gauges and thermocouples were positioned at the pre-decided critical locations of the frame sub-assemblage before the casting of the concrete. High temperature strain gauges as well as ambient temperature bondable strain gauges were mounted on the steel reinforcement with appropriate heat insulation at 154 different locations of the frame sub-assemblage. Furthermore, 301 K-type thermocouples were embedded into the concrete to capture the thermal profile inside different members. Some mineral insulated (MI) steel type thermocouples were also positioned inside the fire compartment. The secondary independent steel frame was installed for mounting the long stroke length Linear Variable Displacement Transducers (LVDT) at different locations of the test frame. The imposed cyclic lateral loads on the test frame were measured using pressure sensors. A total of 491 channels were individually programmed to log their respective data. All the instruments and sensors were checked for proper functioning prior to start the test.

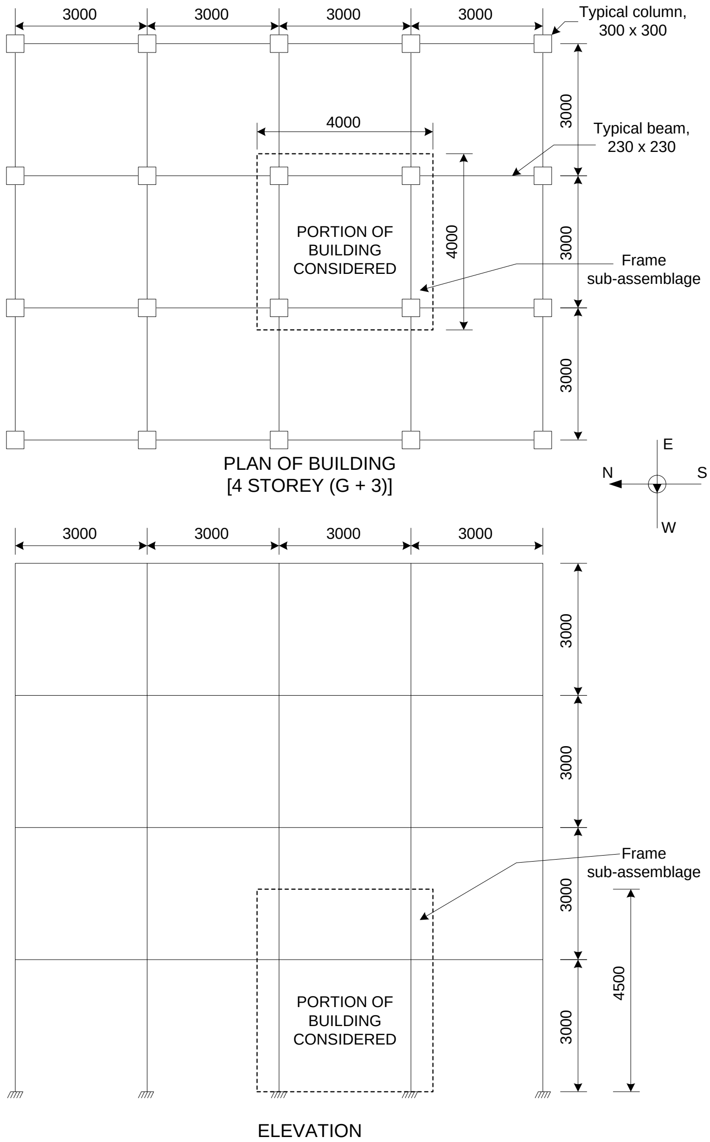
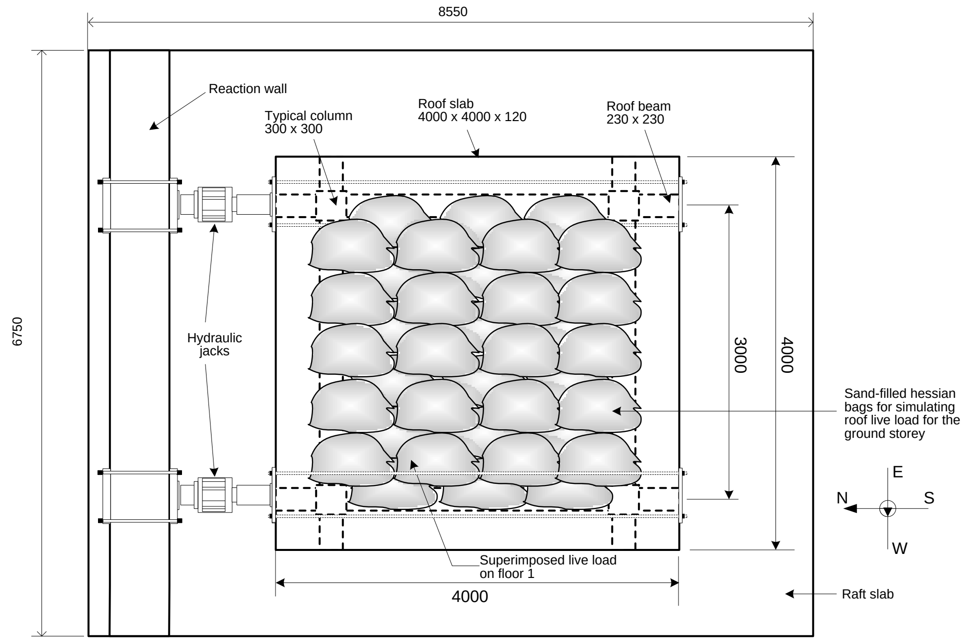
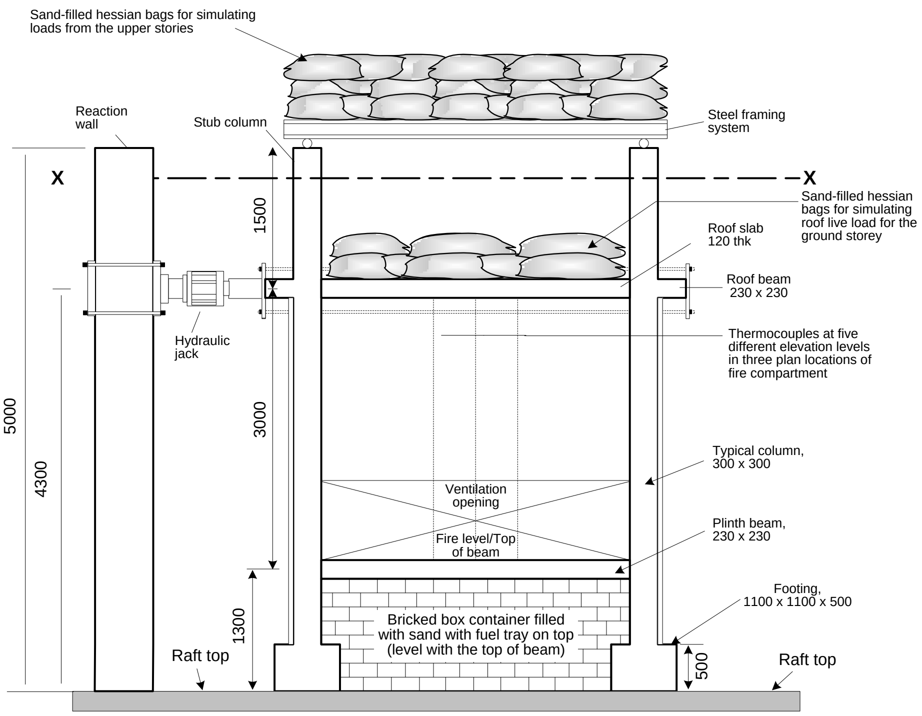


Figure 1. Building configuration and the frame sub-assemblage



(a) Plan of the test setup (at X-X)



(b) Elevation of the test setup

Figure 2. Schematic of the loading arrangement of the test frame

2.3. Lateral Loading

Cyclic lateral loads on the test frame were applied in a quasi-static manner in load-controlled mode using a pair of double acting hydraulic jacks. Push and pull mechanism as shown in Figure 3, was imparted. The lateral load in each cycle was 10 mm and incremented by 10 mm in successive loading cycles. In this way of loading, a final target displacement of 76 mm was induced as mechanical pre-damage to the test frame, and that corresponded to Life safety of structural performance per FEMA [FEMA356:2000]. The similarity between the measured loading and displacement histories of the test frame can be seen in Figures 4 (a-b).

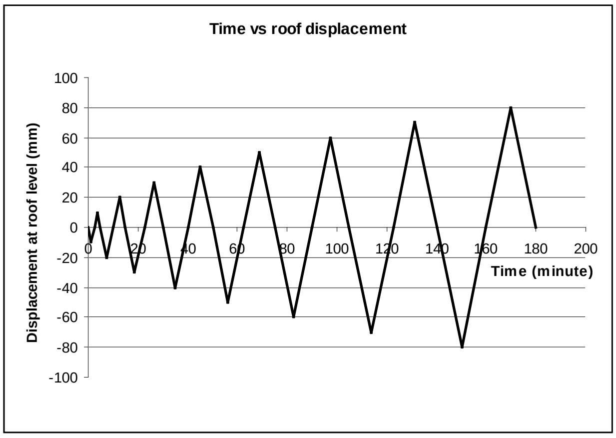


Figure 3. Proposed loading history of the test frame

3. MECHANICAL LOADING TEST RESULTS AND DISCUSSION

The development of cracks and displacement was monitored through dedicated instrumentation in the sub-assemblage of the test frame. No surface cracks in the sub-assemblages were observed upto a roof level of displacement 20 mm to the test frame. The first crack was seen near the joints of the roof beams oriented along the lateral loading direction N-S at a roof level displacement of 35 mm and that corresponds to a base shear of 185 kN as shown in Figure 5(a). Several cracks were observed at different locations of the sub-assemblage in the successive cycles of lateral loading. Some flexural cracks in the columns (Figure 5 (b)) were visualized near the joints of the plinth beam at a lateral displacement of 52 mm. Concrete spalling near the joints of the roof beam spanning along the N-S direction was observed at a roof drift ratio of about 2 %. The test was terminated at a roof displacement of 80 mm corresponding to a base shear of 267 kN. The base shear-roof displacement relationship for the test frame plotted in Figure 6(a) shows no deterioration in the lateral strength of the frame at Life Safety structural performance level. Figure 6(b) shows the hysteresis curve of the test frame which shows that a maximum displacement of 95 mm in the "push" cycle and 85 mm in the "pull" cycle was registered corresponding to base shears of 316 and 267 kN respectively. Stiffness degradation and pinching of the hysteresis loops between successive cycles of loading is seen in Fig. 10 though the hysteresis loops continued to remain stable without any significant degradation in strength. The results of the strain analysis show that plastification of the test frame was initiated before "Life Safety" structural performance level. The test frame did not suffer any significant structural damage.

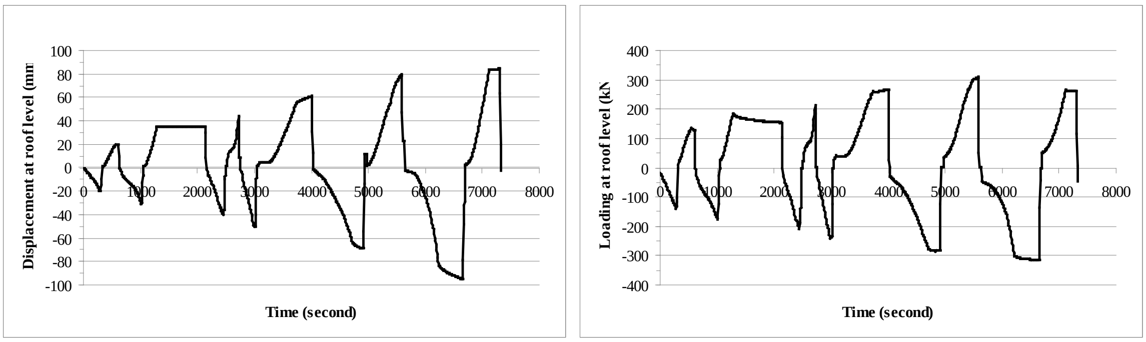
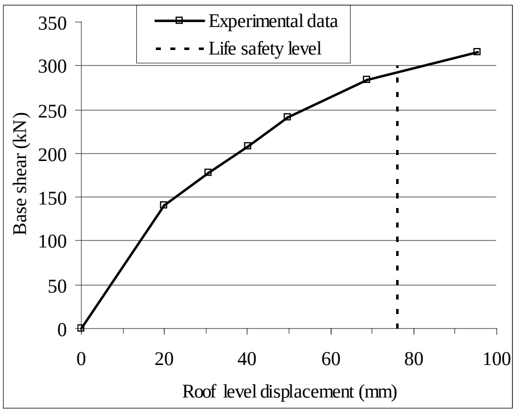


Figure 4. The behaviour of the test frame (a) Measured displacement (b) Measured loading

The typical measured strains in rebars of the roof beams B5 at the critical locations during the entire loading history along N-S direction are shown in Figure 7. It may be noted in Figure 7 that although the rebars being monitored for strain underwent a complete reversal of forces under the effect of the applied cyclic loading, the surface mounted strain gauges were unable to record the compression strains and hence only the tensile strain values were accurately available for analysis. The trends in the variation of tensile strains in Figure 7(a-b) are basically similar to the measured load and displacement behaviour of the test frame presented in Figure 4(a-b). The peaks in the measured load and displacement values during the "push" phase of cyclic loading in Figure 4(a-b) are seen to approximately coincide on the time scale with the observed peaks in the tensile strains plotted in Figure 7(a-b). First yielding of rebars in beam B5 at left end joint was observed at a roof displacement of 52 mm. The results of the strain analysis show that plastification of the test frame was initiated before "Life Safety" level of structural performance. However, the test frame did not suffer any significant structural damage.

Figure 5 (a). Tension cracks in roof beam B7 (b). Flexural cracking in a column C1



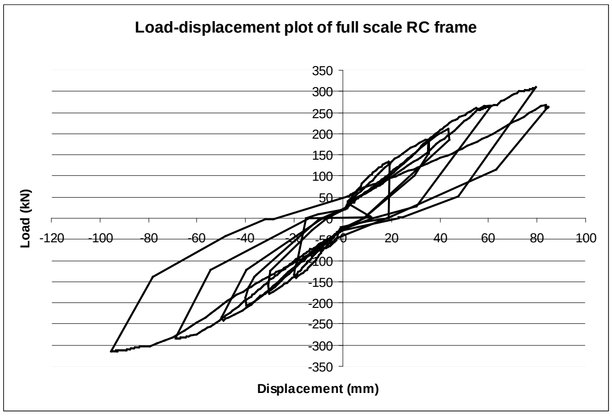
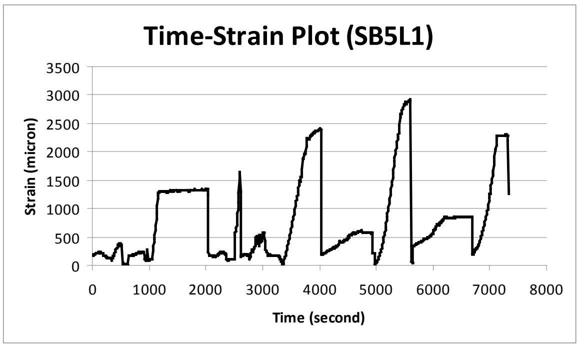


Figure 6(a). Base shear vs roof displacement (b) Hysteretic curve of the test frame



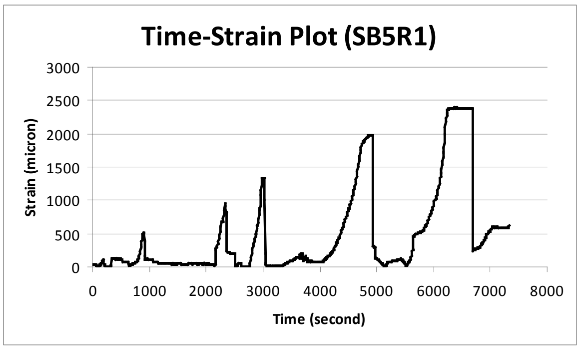


Figure7. Measured tensile strains in rebar of Beam B5 (a) near the top face (a) near bottom face

4. TEST RESULTS AND DISCUSSION OF FIRE TEST

Before exposing the pre-damaged frame to fire, mock fire tests were conducted in a brick masonry fire compartment. Figure 8(a) shows a typical gas temperature history recorded inside the fire compartment during one hour fire followed by a cooling phase. Figure 8(b) depicts the nomenclature used for identifying the locations in the RC frame. However, a gas temperature of above 1000 °C was attained within 5 minutes after ignition of fire indicating the flashover with a maximum temperature of 1289 °C as shown in Figure 9(a). The sound of concrete spalling off from the roof slab was heard after 5 minutes of ignition, which continued for another 15 minutes. The spalling sound corresponded to a compartment temperature between 300 to 400 °C. The structure showed no signs of collapse during and after the fire test. Unfortunately, the data acquisition of vertical displacements at various locations of slab became unsuccessful due to instrument malfunctioning in a high temperature environment. A maximum vertical residual displacement of 46 mm was later measured in the roof slab with the help of leveling. Figure 9(b) shows the beams and columns after the fire test and depicts the signature of the structural stability exhibited by the RC frame sub-assemblage after the fire test. Extensive damage was seen in the roof slab in terms of spalling of concrete and resultant exposure of reinforcement. This was mainly due to the movement of fire plume towards the ceiling causing a rapid flow of hot gases in the radial direction termed as a ceiling jet. Despite this, the test frame withstood one hour fire exposure without losing its structural integrity in terms of complete collapse.

4.1 Temperature Histories

Temperature recording gauge points measured from the exposed surface of the sub-assemblage were located as follows: plinth and roof beams: 5 mm, 25 mm, 115 mm, 205 mm and 225 mm, columns: 5 mm, 40 mm, 150 mm, 260 mm and 295 mm and roof slab: 5 mm, 30 mm, 60 mm, 90 mm and 115 mm.

Figure 10(a) shows typical temperature distribution for column C2 at the support sections and mid-height section. The maximum temperatures recorded at different depths of the top-height section of the column C2 along N-S direction were 862, 579, 135, 92 and 84 °C after 52, 54, 154, 284 and 282 minutes of fire initiation, respectively. Degradation of concrete was prominently observed at locations that experienced high temperature levels (above 600°C). Typical temperature distributions in roof beam B8 are shown in Figure 10(b). The peak temperatures were 900, 626, 202, 123 and 118 °C at the left-span section after 58, 60, 144, 282 and 296 minutes respectively. The maximum temperature recorded was about 900 °C at the surface whereas the temperatures at the mid-depth were not more than 202 °C. Hence, the temperature variations across the depth of the beam section at right end section were quite appreciable. Apparently, the roof beam suffered a maximum degradation of the concrete which was clearly evident during the visual inspection. However, despite that all the roof beams and columns remained intact and structurally stable. Figures 11(a) shows typical temperature profiles of roof slab at the points located along North-side near the window opening. The peak temperatures obtained were 1038, 575, 333, 296 and 252 °C at 56, 60, 72, 116 and 128 minutes, respectively. Considerable thermal gradients were obtained at many locations in the roof slab, which explains the reason for severe spalling of the cover concrete in roof slab. Despite the considerable spalling, roof slab remained structurally intact and continued to carry superimposed loads.

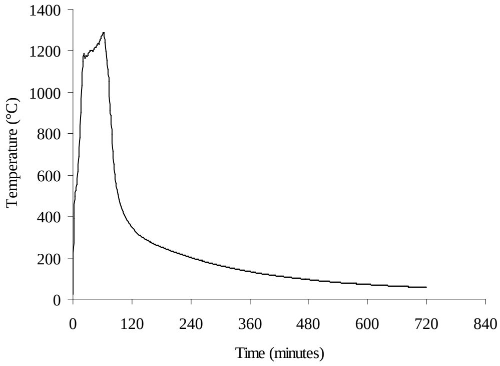
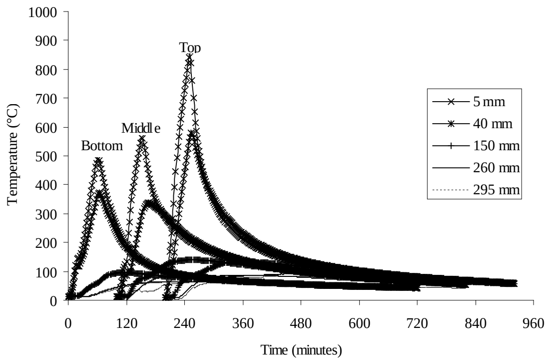


Figure 8 (a). Time-temperature plots for the compartment fire (b) Sub-assemblage nomenclature

Figure 9. (a) Fire test in progress (b) Frame sub-assemblage after fire test



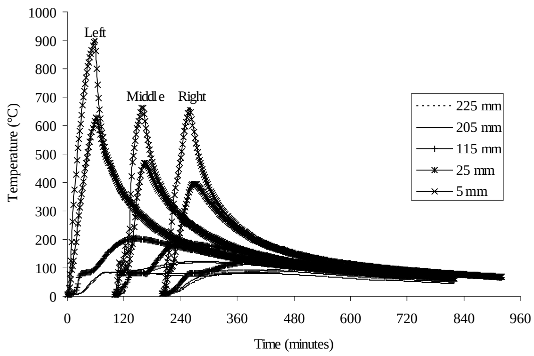


Figure 10 (a). Time-temperature curves for column (b). Time-temperature curves for roof beam

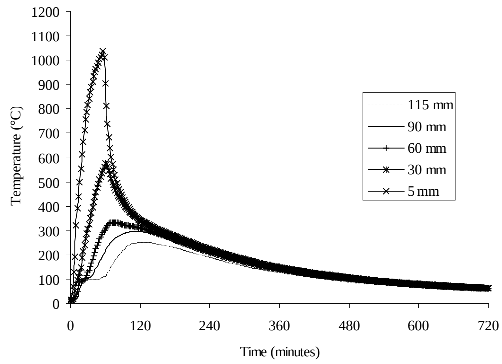


Figure 11 (a). Typical time-temperature curves for roof slab

Analysis and interpretation of strain data was a complex task. As expected, many strain gauges malfunctioned at high temperatures and questioned the accuracy of obtained values. However, the results of measured strain at few locations were very encouraging and traced the behaviour of the frame sub-assemblage in fire.

5. CONCLUSIONS

Testing of a monolithically constructed RC frame sub-assemblage under quasi-static lateral loads i.e., simulated earthquake loading followed by fire loading has been presented. As intended, the test frame was able to withstand a damage corresponding to 'life safety' structural performance level without collapse. First sign of cracking was observed in the beams followed by that in the columns. It was seen that the highest temperature attained in the concrete in sub-assemblages of the test frame was 1038 °C in the roof slab. The sound of concrete spalling off from the roof slab was heard after 5 minutes of ignition, which corresponded to temperatures between 300 to 400 °C. Failure of the displacement transducers during the fire test hindered the understanding of real time vertical displacement of the roof slab and an overall structural behaviour of the test frame. Thus, the full-scale testing of the RC sub-assemblage successfully justified the structural stability of the frame under fire followed by a quasi-static lateral cyclic mechanical loading.

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